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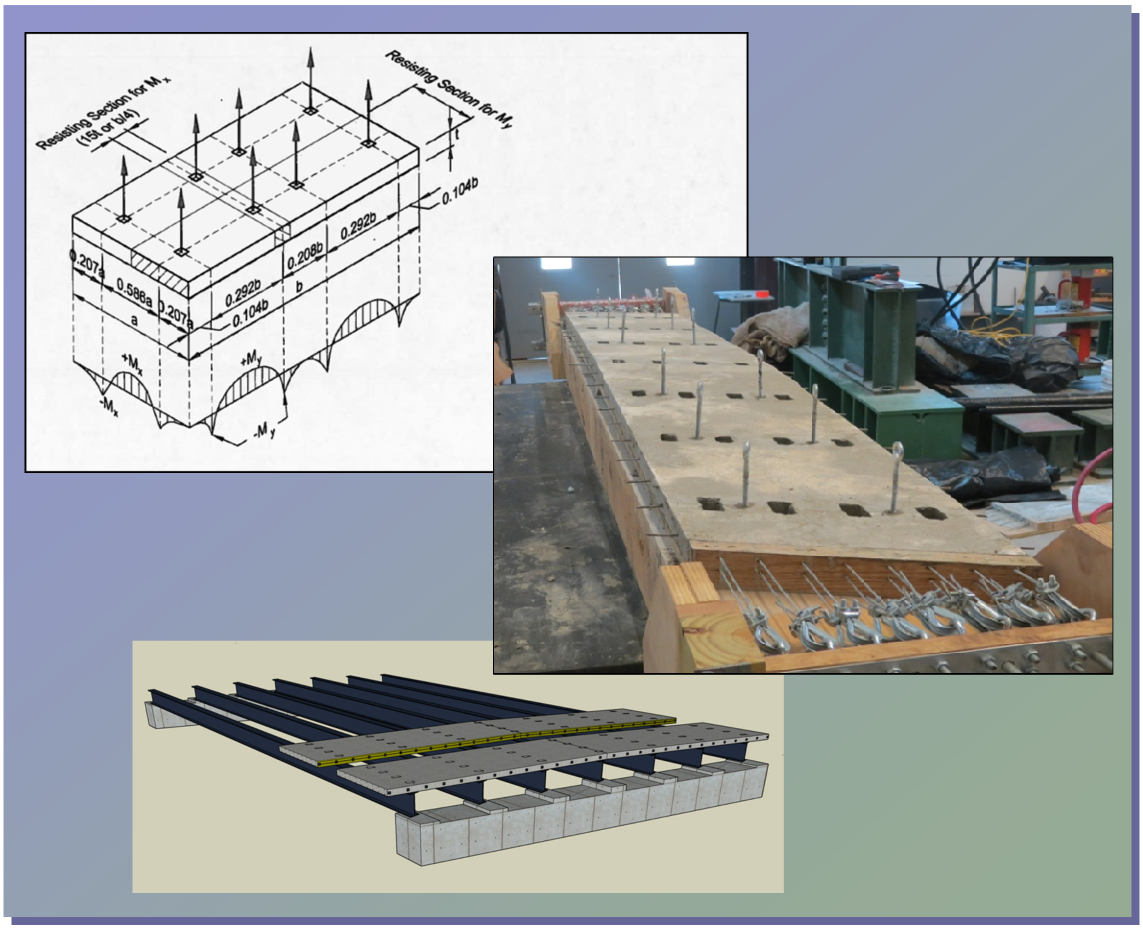

The Future of Rapid Bridge Deck Replacement

Final Report

Prepared by the University of New Hampshire
Department of Civil and Environmental Engineering for the
New Hampshire Department of Transportation in cooperation with the
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16. Abstract

Replacing aging, deteriorated infrastructure often requires road closures and traffic detours which impose inconvenience and delay on commerce and members of the motoring public. Accelerated bridge construction techniques often use precast members to reduce the duration of construction and the resultant disruption to traffic patterns.

This project considered various technical, safety, and economic challenges in the design and construction of a full-width precast concrete slab with a centerline crown. Torque verified leveling screws compensated for girder, deck panel prefabrication tolerances, and dead load deflections. In addition, the uniform-depth deck slab was cast by conventional prestressing fabricators. The method to lift and transport the slabs was considered to prevent damage. Accelerated bridge construction is less expensive and bridge work can be completed and the bridge reopened to traffic within a matter of days versus months for conventional deck replacement.

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The Future of Rapid Bridge Deck Replacement

University of New Hampshire

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It is common knowledge that a large portion of the nation's highway infrastructure is deteriorating to a severe level. In addition to technical challenges, the cost of repairs and associated safety issues extend beyond the basic construction to neighboring business and area travelers. State Departments of Transportation bridge rehabilitation inventories are increasing, at the same time budgets are being difficult to expand and in many cases difficult to sustain. Accelerated Bridge Construction, ABC, offers many cost reducing alternatives, a concern is also reducing the impact of traffic detours imposed on the commercial and motoring public. Creative detours utilizing existing site ramps and other feeder roads need to become a higher priority when planning bridge repair projects.

Minimizing this impact to travelers is one of the biggest challenges in bridge deck replacements. With more than 67,000 of the nation's bridges approaching the end of their design lives, the need for a quick and economical method to replace bridge decks is increasing rapidly. One of the more recent techniques being utilized for single span bridges is installing members, for example box sections longitudinally with traffic direction and slab sections perpendicular to traffic direction. This lessens the time needed to form, pour, and cure a cast-in-place deck; the panels and box sections require their adjoining longitudinal joints be bonded, a closure pour, with a high strength and fast setting adhesive/sealant. The curing time required for these joints can extend the bridge closure period. There is a need to reduce the number of panel connections and accelerate joint adhesive/sealant curing (see figure 1).

The location of these cold joint closure pours can also cause serious problems for example:

1) Closure pours transverse to traffic flow – These pours along the entire deck member length must transfer shear between adjacent precast members spanning across the width of the bridge. Shear between adjacent deck members approaches zero over supporting girders and maximum at midpoints between supporting girders. These deck member midpoint locations are also at maximum positive bending points resulting in high principle stresses.

2) Closure pours longitudinal to traffic flow - These pours along the entire length of the bridge must support shear between adjacent longitudinal members and the effects of maximum positive moment at the longitudinal midpoint for these members.

Locating pours in areas of such high induced stress warrants concerns about the bond strength developed in fast setting concrete. A substandard bond will lead to eventual water infiltration into the joint, which will most likely compromise the joint reinforcement, resulting in a reduction in service life. Cracks forming along closure pours in maximum negative regions occur in the top of the bridge deck members at for example girder support locations and accelerate water penetration into the joint resulting in the impervious layer debonding from the closure pour. Concrete closure pours also add time and on-site labor costs to the project. The full closure pour curing time normally exceeds Accelerated Bridge Construction (ABC) times. There is a need for improved joint designs and adhesives/sealants.

Wearing surface transverse slopes pose a challenge for full width precast deck replacement projects. Steel girders supporting cast in place concrete decks were typically designed for a uniform deck thickness; girder supports were set at elevations to yield desired cross slopes. Steel girder fabrication tolerances, camber, and decking dead load deflections were accommodated in part by varying haunch thicknesses. These procedures assured that the design vertical deck profiles were maintained. An ABC procedure is needed to accommodate residual girder camber after deck removal and cross slope designs.

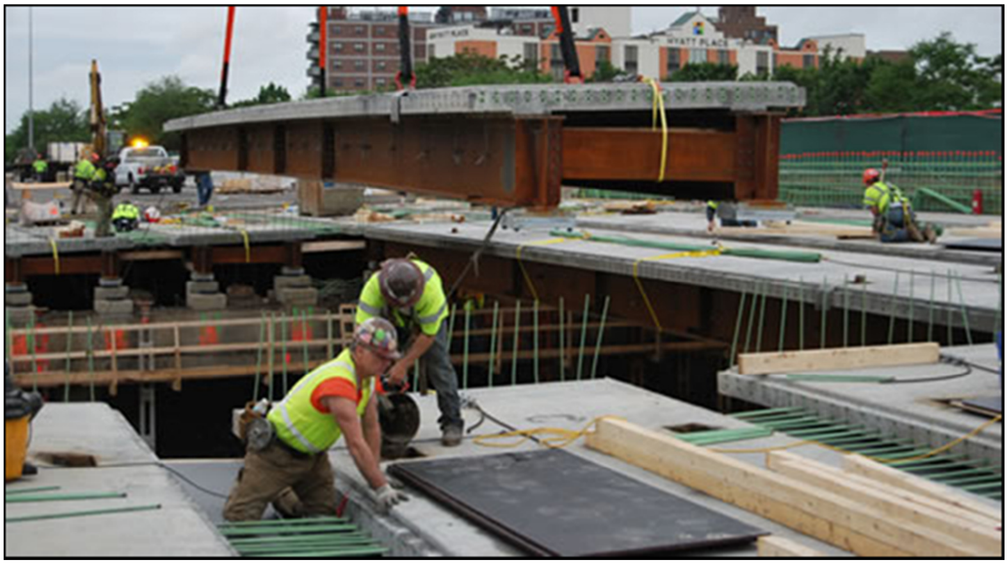

Figure 1: Zipper Pours - Massachusetts Fast 14

This research considers these technical, safety, and economic challenges. A full width, precast concrete slab designed with a centerline crown with edges at a 2% slope away from that crown is investigated. The use of torque verified leveling screws are used to compensate for girder, deck panel prefabricated tolerances, and dead load deflections. Concerns with prestressing a crowned slab of uniform depth and shipping/handling issues are addressed.

Overview of Research

The overall ABC research goal is to make full bridge width, constant depth, crowned prestressed deck slabs with post-tensioning ducts, stud pockets, and tongue and grooved transverse joints installable using accelerated bridge construction best practices. Slab design constraints include ACI code guidelines, a conventional casting bed, conventional lifting apparatus for handling, and transportation alternatives. This research includes fabrication details, handling, and shipping strategies.

Other areas that address service life for full width crowned slabs designed for ABC installation alternatives include shear key designs, presetting leveling devices, and segmental longitudinal post-tensioning, which were concurrently being researched. The shear key design joining two adjacent slabs prevailed using a fast setting polymer to seal the joint and post-tensioning bars to transfer load. (see Figure 2) Leveling screw research resulted in an analysis procedure to determine leveling screw heights to achieve the design bridge vertical profile of the fully installed bridge deck. The ABC construction steps are setting the slab with preset leveling screw lengths, applying the joint polymer sealant to the adjoining surfaces, pushing the slabs together to squeeze out excess adhesive, post-tensioning the bars when the adhesive reaches 500psi, install the studs, grout the haunches and stud holes, and inject a sealant into the post-tensioning ducts.

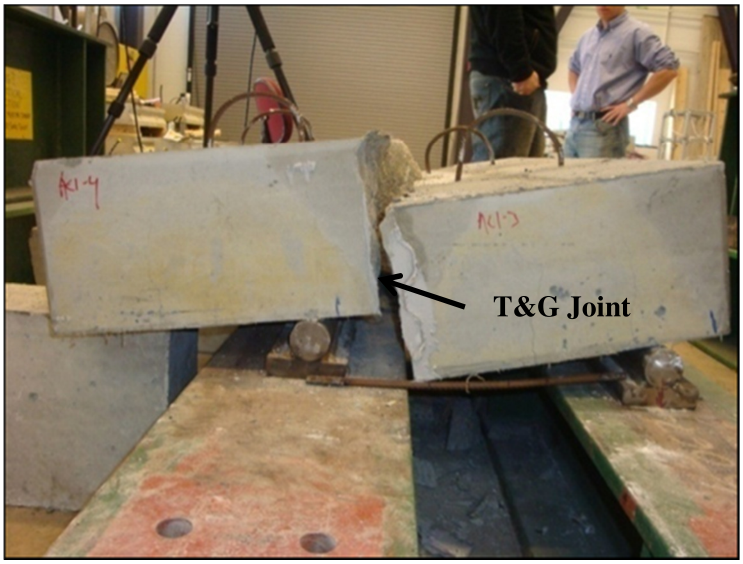

Figure 2: Shear Key Design Lab Test

NHDOT bridge designers and professors at the University of New Hampshire worked with graduate students to research all innovations that can be implemented using cost effective labor and conventional equipment within all conventional construction safety practices. The research resulted in a near full scale model to demonstrate all construction steps. A trial build was videoed and presented to contractors bidding a 70 foot by 48 foot deck replacement project funded under the FHWA Highways for Life program to be rebuilt in 60 hours and opened to traffic.

Demonstration of Research

The demonstration project is the US Route 3 overpass to NH Route 11A with two 12 foot traffic lanes,

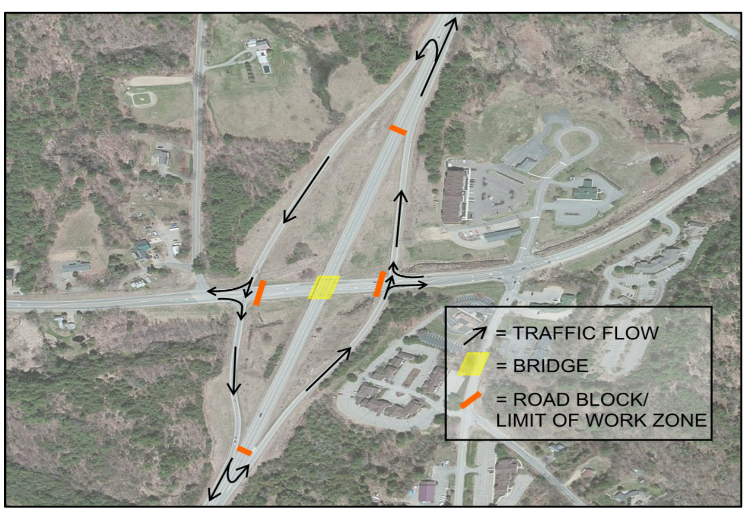

Figure 3: Traffic Pattern During Construction

10 foot shoulders and a 23 ° skew. While the bridge deck has a rating of 3 for structural deficiencies, an inspection of the superstructure revealed no deficiencies sufficient to restrict a bridge deck replacement for full restoration. The project, as proposed to the FHWA, defines a 60 hour road closure; surrounding municipalities accepted the closure as the average daily traffic during the construction season is 10,000 and the closest detour is 15 minutes. To minimize the detour a roundabout concept using the four ramps connecting the two roads was proposed and accepted (see Figure 3). The ramp layouts divert traffic and leave more than 1,000 feet of closed road space along Route 3 for construction use.

The deck replacement will consist of 9 slabs installed segmentally along the length of the bridge (see Figure 4). The slabs are 57 foot long from tip of skew to tip of skew, 8 foot wide, and 9 inches thick. There are 7 rows of 4 blockouts each for shear studs. These are located where the slab will sit on the existing girders. The slabs will have the shear key design cast along their longitudinal sides. A diagram of the first two slabs being set into place is shown in Figure 4: Gilford Bridge.

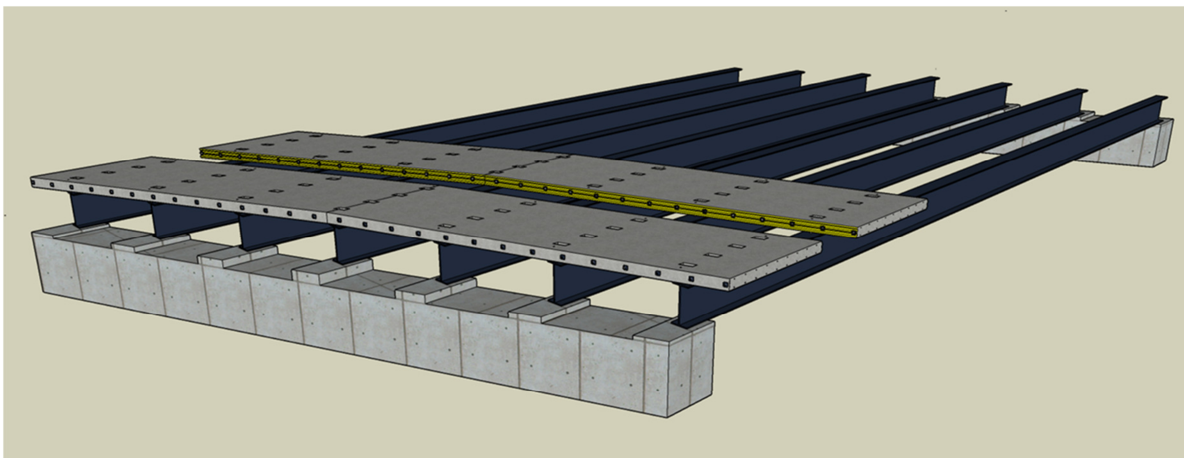

Figure 4: Gilford Bridge

Research Techniques

The conclusions of this research were reached with a variety of techniques. First and most basic was a series of manual calculations. Simple statics, beam analyses, and strength of materials principles were applied to different aspects of this research. Programs such as Microsoft Excel and MathCAD were used to perform calculations and verify accuracy. SAP2000 was used to create finite element models to analyze full scale computer models and compare laboratory experimentation results to the lab size model.

A 1/8 scaled model was built in the University of New Hampshire lab. The model was designed to include all the same characteristics of the full size slabs that will be used on the Gilford Bridge. These include the same 23° skew, the crown, the same number of prestressing cables, and the same number of skewed blockouts for shear studs. Multiple concrete mix designs were tested prior to the construction of the model. This model was then used to test the overall casting methodology for creating a prestressed crowned slab, determining lifting locations, and testing a model transportation support.

Casting Prestressed Crowned Slabs

Casting crowned deck slabs requires prestressing cables to be lifted at the crown, the inverse of draped cables being pulled down. Eight strands were placed in the top half of the slab as well as the bottom half of the slab for a total of 16 cables, concentric about the neutral axis of the slab to induce uniform prestressing and prevent camber. Casting a crowned uniform thickness slab duplicates the constant thickness of the original cast-in-place deck. The neutral axis of the slab, including the prestressing steel, remains at mid height of the slab from end to end except at the very middle. This slight offset due to eccentricity in the slab is so miniscule it has no detrimental effects to the overall concentric stressing. (See Figure 9: Cross Section at Crown) The crowned slab can be installed on existing girders to preserve the original bridge longitudinal and vertical profiles with reasonable haunch thickness. Figures 5 & 6 show the difference between installing a crowned slab versus a flat slab with a crowned asphalt surface. With a 2% transverse slope from the crown, the vertical difference between the edge and crown elevations of the Gilford Bridge is nearly 6 inches. This adds up to nearly 16 ft^3 of excess material per longitudinal foot along the length of the bridge. It would not be feasible to replace the existing deck with flat precast slabs and varying wearing surface and haunch thickness due to exceeding dead load capacity, and it would not meet the existing approach ramp longitudinal and transverse profiles.

Figure 5: Flat Slab with Extra Material

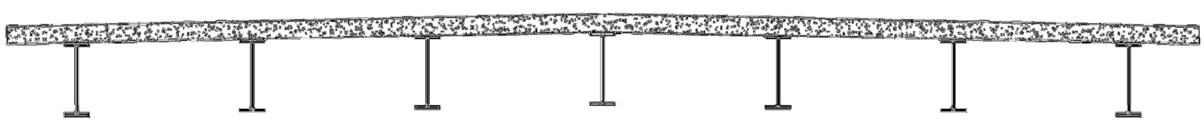

Figure 6: Crowned Slab

SAP 2000 Model - Concentric Behavior of Crowned Slab

A model of the full scale Gilford Bridge slab was created in SAP2000 using solid elements. To test the prestressing in the slab, support conditions have to allow for any displacement in the x, y, and z directions while remaining stable. The analysis is intended to confirm the performance of a concentrically crowned slab. End supports were eliminated in this specific analysis. Dead load was not analyzed, just prestressing forces. Supporting the middle along the crown with x, y, and z constraints allows the ends of the slab to displace if the prestressing causes camber; both ends of the slab would essentially be free floating.

Four pin supports were modeled at the middle of the slab where the crown came to a peak. Two pin supports on either side of the bottom face of the slab, and two more on the upper face. Supports on both sides were necessary to keep the slab from rotating around any single axis. These support conditions can be seen in Figure 7: Support Locations in SAP20007 & 8.

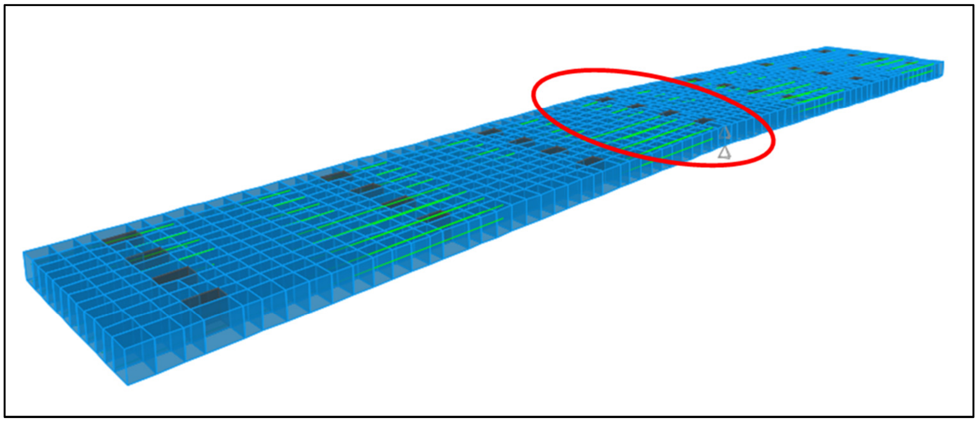

Figure 7: Support Locations in SAP2000

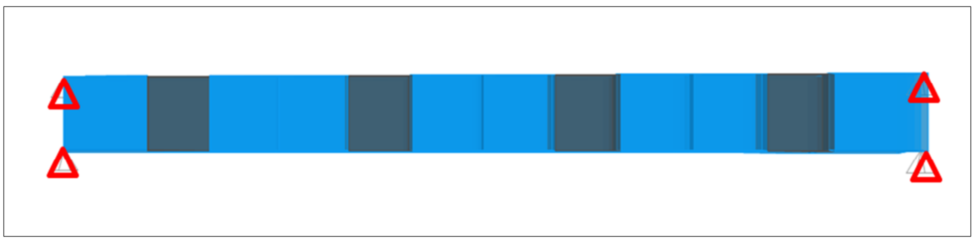

Figure 8: Cross Section of Support Locations

Camber Analysis Results

The analysis showed that the prestressing forces caused only 0.0014 inches of deflection at the far ends of the slab. This minimal amount of camber was expected due to the eccentricity of the cables at the crown. Figure 9 shows the depth of concrete and cable locations at the crown. There is no way to place constant thickness tendons concentrically through the crown vertex in the slab, however, a camber of less than 1/16" is well within the acceptable limit for full scale deck slabs.

UNH Lab - Casting the Lab Model

Reinforcing

One quarter inch wire mesh was used to reinforce the 1/8 scale lab model. Two layers of this mesh were placed throughout the slab. The top and bottom meshes were both about 1/4" from the top and bottom surfaces. The mesh was tied to the prestressing cables to keep the vertical spacing of the two layers even. As seen in Figure 10 metal rods were laid transversely on the skew of the slab. One layer of these rods was placed concentrically throughout the slab. These rods were placed to simulate transverse deck post-tensioning to be installed in the replacement bridge.

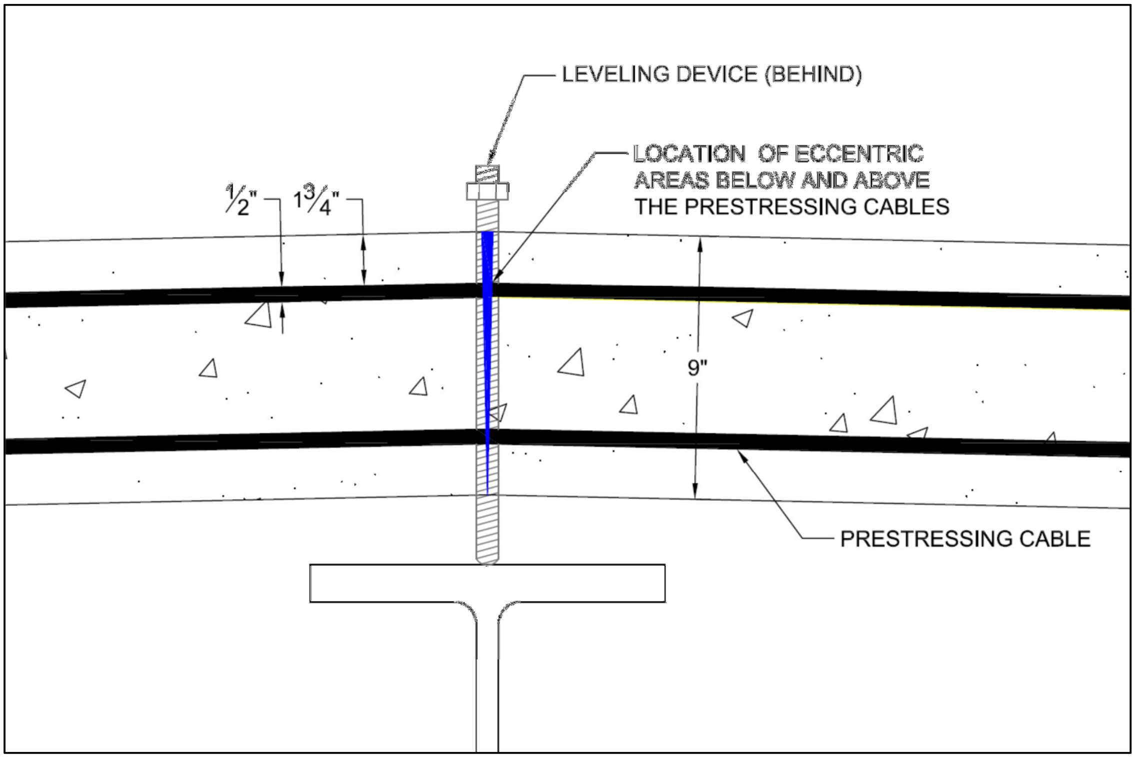

Figure 9: Cross Section at Crown

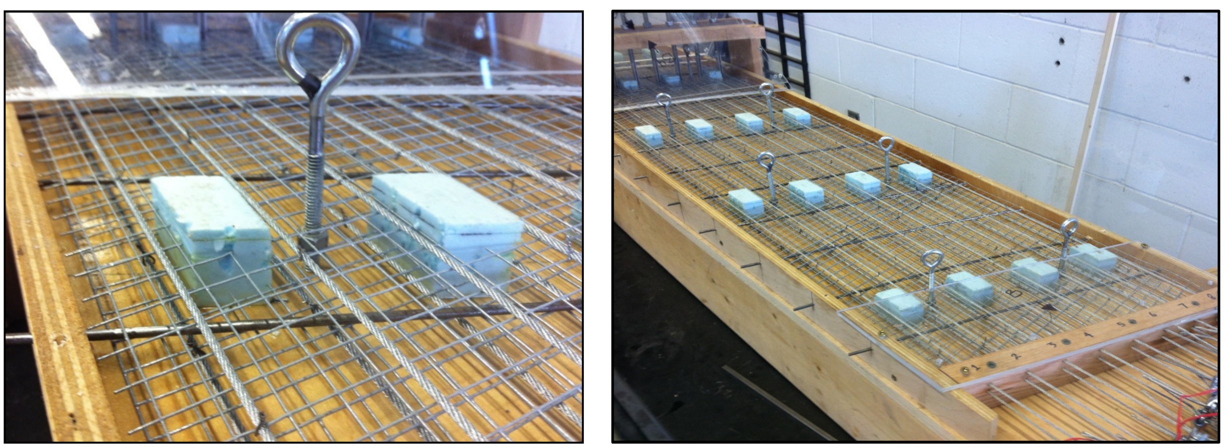

Figure 10: Lab Model Reinforcing

Prestressing

One eighth inch diameter wire cables were used to prestress the model. Each cable was pulled through a pre-drilled hole in the form to ensure even spacing horizontally and vertically. At the crown, the cables were tied to mini rollers connected to an overhead beam supported to elevate the cables. This setup is shown in Figure 11: Prestressing Cables at Crown

Figure 11: Prestressing Cables at Crown

The prestressing cables were tied to eye hooks at each end of the form. The eye hooks at one end were used to stress the cables and the eyehooks at the other end stayed in place. Two strain gauges were applied to opposite sides of the immovable eyehook screws (See Figure 12). Doubling the gauges helped ensure accurate strain readings and compensated for any eccentricity in the eyehook. Both gauges were measured and then averaged. The readings from the gauges were used to measure the force applied to the prestressing cables. Each cable was pulled to 400lbs, simulating 30ksi in a full scale bridge deck panel.

Figure 12: Strain Gauges

Pulling 16 cables with 400lbs of force induced 483 psi of compressive stress in the cross sections where the blockouts are located and 410 psi of compressive stress in cross section with no blockouts. The blockouts are place along a cross section drawn parallel to the member end 23° skew. A similar increase of 15 to 20% will occur in bridge deck slab designs when placed at a skew to the supporting girders; the larger the skew angle the lower the stress increase.

Self Consolidating Concrete (SCC) Mix

The SCC mix design contained two admixtures: a superplasticizer and a shrinkage reducing agent. Multiple batches of the mix were tested prior to casting the model. The final mix contained a variety of sands that were sieved to achieve a desirable gradation and maximum grain size. The size of the sand went up to a number 30 sieve. The mix can be found in Table 1: SCC Mix Design. The final compressive strength of this mix after fully cured was 9400 psi.

Table 1: SCC Mix Design

SCC Mix for Lab Model		
Material	**2 Cubic Ft Mix**	**By Weight**
Fine Sand	45.2 lbs	21%
Sand up to #10 Sieve	10.0 lbs	5%
Sand up to #20 Sieve	45.2 lbs	21%
Sand up to #30 Sieve	15.0 lbs	7%
Type I/II Portland Cement	70.8 lbs	33%
Water	26.2 lbs	12%
Superplasticizer	360 mL	-
Water Reducing Agent	240 mL	-

Placing the Concrete

The form was built with 2% incline to the crown, installed at a 23 degree skew to the longitudinal slab direction making it parallel to the slab ends. Plexiglass was cut into sections to be attached to the top of the form to assure constant slab thickness. The concrete mix was placed starting at one end of the slab, seen in Figure 13: End of Slab FormLabel “A1” is shown on the first plexiglass sheet shown in Figure 14. Sections of the plexiglass were attached as the concrete was placed to facilitate compaction and to sustain constant thickness. A continuous flow of self consolidating concrete was maintained to minimize the possibility of air bubbles forming in the mixture.

Figure 13: End of Slab Form

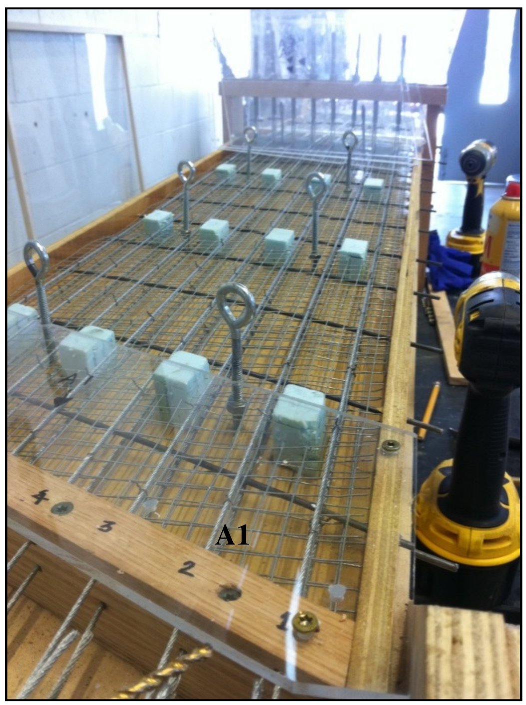

Figure 14: Full Slab Form before Concrete Placement

Stripping

The form was stripped from the slab 48 hours after casting (seed Figure 15), though not lifted or moved at this point. The plexiglass and surrounding sides of the wooden form were removed. The slab was kept moist for one week.

Figure 15: Full Slab after Stripping

At 7 days the slab was symmetrically stressed as shown in Figure 16.

Figure 16: Releasing the Prestressing Cables

After the slab was lifted and set back down on the form, it was observed that the ends of the slab had cambered slightly upward on the far obtuse ends. About a ¼" of permanent camber was measured at each end from the form to the bottom of the slab. This camber is due to the top plane of 8 cables being placed with approximately 1/16" of eccentricity relative to the lower cable layer about the neutral axis of the slab. This can be seen in Figure 17.

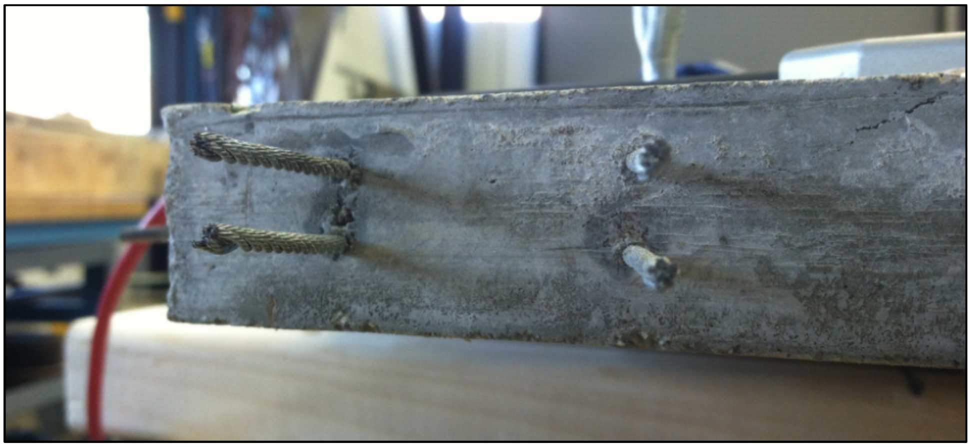

Figure 17: Eccentric Prestressing Cables

The camber in this model proves that the prestressing was successful as well as the importance of placing the prestressing cables concentrically throughout the slab. Even a minimal amount of eccentricity in cable placement has an exponential effect on the amount of camber. This was confirmed with computer analysis using the SAP2000 models. On a full scale slab, it was determined that just 1/16" of eccentric placement of the prestressing cables would lead to ¼" camber in the far ends of the slab, as was seen in the lab model.

Lifting of Slabs

The Precast/Prestressed Concrete Institute Manual's lifting guidelines are based on solid rectangular concrete slabs with a constant thickness and flat profile. The slab design for this research project does not meet all of these criteria. The slabs for Gilford are skewed, crowned, and have four stud blockouts in a cross section reducing the area by one third. Additionally, one of the objectives with these slabs is to use the leveling devices as lifting hooks as well as supports during installation, though this restricts the number and location of lifting hooks to the locations bearing on the support girders. The PCI Manual designates ratios to locate lifting points but does not account for slab designs in which the ends, blockouts, and lifting hooks are placed at a skew. To meet the PCI's lifting hook location equations would require lifting hook inserts, resulting in additional reinforcement.

Leveling device locations depend on the spacing of the bridge support beams. It is imperative that two leveling screws be installed per bridge support girder. Typically, bridge steel girder spacing does not exceed approximately 8 feet. The devices must be located to match the spacing exactly to the field conditions, and for Gilford this is every 7'-4".

SAP2000 – Lifting the Gilford Bridge Model

To test the various lifting techniques, the same SAP2000 model that was used to test prestressing camber was reconfigured and used to test lifting. This model was created to exactly replicate the Gilford Bridge slabs; however, the slab configuration for the Gilford Bridge project has characteristics that add an additional level of challenge for design, analysis, and construction. The slabs must be constructed with ends and crown at the same skew. Due to the extra length, crown, lifting/leveling devices, and blockouts for the shear studs, it was determined finite element analysis would be best for these bridge deck slabs. SAP 2000 solid multiple sized finite elements were used; Excel was used to interactively edit the model nodes.

Concrete strength was specified as 8 ksi, the thickness, width, length, skew, crown, were all modeled to represent the Gilford Bridge deck. Sixteen prestressing cables are modeled with eight equally spaced tendons, 2" from the slab bottom and eight similarly below the top. Tendons were used to model the cables as only axial forces and displacements are needed. The cables were 1/2" in diameter with an ultimate strength of 64 ksi. The cables were stressed to 27,000 psi with a node entered at the longitudinal midpoint of the slab to sustain concentric cable placement along the longitudinal slab length; the slabs are cast with a 2% slope from the skewed crown.

Varied Length Models

Three additional models of varying lengths similar to the Gilford Bridge model were prepared to evaluate limits of the PCI lifting alternative configurations. Using common points for lifting and leveling restricts the lifting points to girder locations where the blockouts and leveling devices are required. The longitudinal block out spacing on each slab model was held to the typical girder spacing of 7'-4" as required for the Gilford Bridge model. The details of each slab are tabulated in Table 2: Varied Length Models.

Table 2: Varied Length Models

Slab Length	Rows of Blockouts	Number of Leveling Devices
34 ft	5	10
48 ft	7	14
65 ft	9	18
78 ft	11	22

Concrete Strength at Stripping

Concrete strength at release is assumed to be 75% of the full compressive strength. A further reduction, a factor of safety of 1.5 is taken into account. See Table 3: Concrete Strength at StrippingThese values for tensile strength do not necessarily reflect the limit of stress before cracks will develop. To fully ensure that no cracking will develop when lifting after stripping, it will be best to lift the slabs in a manner that will keep the slab in compression throughout.

Table 3: Concrete Strength at Stripping

Compressive Strength (psi)	75% Compressive Strength (psi)	Allowable Tensile Stress at Stripping (psi)	Safety Factor of 1.5 (psi)
10,000	7,500	650	433
9,000	6,750	616	411
8,000	6,000	581	387
7,000	5,250	543	362
6,000	4,500	503	335
5,000	3,750	459	306
4,000	3,000	411	274

Available Lifting Devices

Leveling devices can be made from threaded rod. The diameter is a function of slab weight, leveling screw spacing, and number of leveling screws serving as lifting hooks. The tributary area of each leveling device should be used to calculate the screw size. Care must be taken to consider the lifting cable angle to prevent bending the screws.

Four Point Lift

PCI has a number of different recommendations for lifting slabs. The first is a four point lift, as shown in Figure 18. This method calculates the first lifting position by multiplying the length of the slab by 0.207 to set the first position distance from the edge of the slab. The same method is used to find the lifting location along the width of the slab.

Maximum negative and positive moments can then be found with the predefined moment equations outlined by PCI, shown below in **Error! Reference source not found.**. These equations are for slabs that have a uniform thickness with no blockouts, yielding a uniform weight.

Eight Point Lift

PCI's recommendation for an eight-point lift includes the same equations for use along the width of the slab. The distance along the length of the slab is determined by two separate equations. To place the first lifting point (outside), multiply the length of the slab by 0.104 and measure that distance in from the edge. The second (inside) lifting point is located by multiplying the length by 0.292 and setting that distance the first lifting point. As with the four point lift, PCI outlines the equations used to find the maximum negative and positive moment when the uniform slab is lifted from those locations.

Gilford Bridge Lifting Calculations per PCI Recommendations

$a := 9ft \qquad b := 48ft$

$t := 9in \qquad w := 112.5psf$

Resisting My Width: $\quad Rywidth := \frac{a}{2} = 4.5ft$

Resiting Mx Width: $\quad Rxwidth := \min\left(15t, \frac{b}{4}\right) = 11.25ft$

Four Point Lift Maximum Moments

$0.207a = 1.863ft \qquad 0.207 \cdot b = 9.936ft$

$0.586a = 5.274ft \qquad 0.586 \cdot b = 28.128ft$

$My4 := 0.0107 \cdot w \cdot (a) \cdot (b)^2 \qquad My4 = 24.961kip \cdot ft$

$Mx4 := 0.107 \cdot w \cdot a^2 \cdot b \qquad Mx4 = 46.802kip \cdot ft$

Eight Point Lift Maximum Moments

$0.104 \cdot b = 4.992ft$

$0.207a = 1.863ft \qquad 0.292 \cdot b = 14.016ft$

$0.588a = 5.292ft \qquad 0.208 \cdot b = 9.984ft$

$My8 := 0.0027 \cdot w \cdot a \cdot b^2 \qquad My8 = 6.299kip \cdot ft$

$Mx8 := 0.0054 \cdot w \cdot a^2 \cdot b \qquad Mx8 = 2.362kip \cdot ft$

Equation 1: Lifting Calculations

PCI Four and Eight Point Lifts for Gilford

Calculations were completed for the proposed Gilford Bridge deck replacement slabs using PCI's recommendations for four and eight point lifts (see Figures 18 & 19). It was assumed the slab was not skewed and had no blockouts, just as the slabs are illustrated in the PCI guidelines. The length, width, and depth of the Gilford slabs were used in the calculations.

Prestress losses were calculated and used to determine the final prestressing stress in the slab. This stress was combined with the stress calculated from the moment that PCI defines. For comparison, SAP2000 analyses of the PCI recommended lifts were completed.

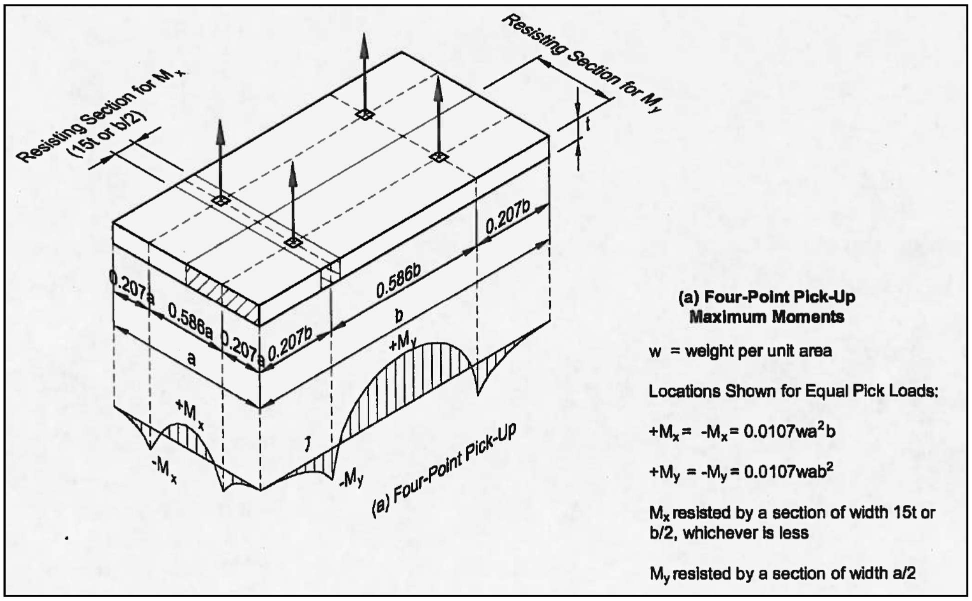

Figure 18: PCI Excerpt - 4 Point Lift

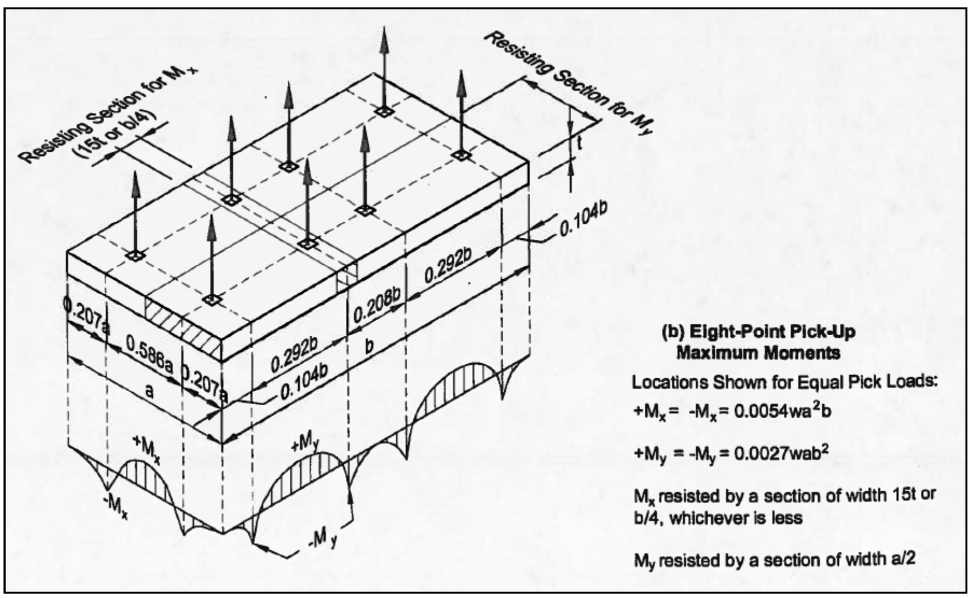

Figure 19: PCI Excerpt - 8 Point

The area and the moment of inertia used in the calculations were taken at a cross section where the blockouts exist. The losses in slabs with blockouts are not consistent throughout the length of the slab, so instead of calculating the losses segmentally along the length of the slab just one conservative cross section was used. This resulted in higher losses and therefore less prestressing force. It was decided to take the conservative approach to eliminate any chance of cracking in the slab due to judgment based on numbers using the solid cross section of the slab.

The stress in the slab due to the PCI defined moment was calculated based on positions along the slab where blockouts exist. This reduces the moment of inertia and results in a higher tensile stress; again, a conservative approach.

PCI recommendations for lifting do not include a defined limit for slab length, so it is difficult to know which lifting scenario would work best on longer slabs. According to calculations, a four-point lift will induce a minimum of 132 psi in tension for the Gilford Bridge. This is not guaranteed to not crack when lifted since there is tensile stress developing within the slab. The maximum tensile stress is found in the middle of the slab along the underside. This would be the worst possible place for cracks to develop since the point of the slab is to eliminate the cold joint in this same region. With the crown this only intensifies this need for no cracking since that is a delicate area with the slope change and the prestressing coming to a point.

With these calculated moments, a four point lift for a Gilford Bridge slab would result in 412psi tensile stress. However, these PCI calculations are for a slab without shear stud blockouts. Typically the shear studs will be located at the longitudinal center of the slab with dimensions of 12" long by 8.15" wide. This changes the cross section of the slab at the point of maximum moment in the y direction (My). The moment of inertia then becomes 0.1104 ft^4 at the point of maximum moment, making the maximum stress at the center 588 psi with blockouts.

In chapter 8 of the Standard Handbook for Civil Engineers, it is stated that that for an uncracked condition of a one-way slab the maximum hypothetical tensile stress is equal to $7.5\sqrt{f'c}$. For a concrete mix design of f'c = 8000psi, the maximum allowable tensile stress would theoretically be 670psi. With a safety factor of 1.5 to limit cracking this bring down the allowable to about 450psi. These maximum allowable tensile stresses were previously outlined in Table 3: Concrete Strength at Stripping for when the concrete has not yet fully cured. The same allowable stresses can be applied to fully set concrete; just use the column of 75% strength as the f'c value.

Positioning the leveling devices in an eight-point lift configuration, the slab will see a maximum tensile stress of 348psi. Combining this stress with the prestressing compression stress, this will be acceptable when lifting fully cured slabs but not when the slabs are just being released from the forms. Lifting from 12 points from leveling devices brings the maximum tensile stress down to 215 psi.

Using Two Cranes

It is assumed that when two cranes are used they are operated simultaneously to symmetrically lift the slab. The lift locations selected must allow low enough tensile stresses that safety factors can be added to account for human errors in lifting. The stress can be due to flexural stress both in the positive and negative areas. The length between lifting points is critical and may differ from PCI recommended locations. Additionally, lifting at full strength compared to at stripping makes a difference where they can be lifted.

Using One Crane

When lifting with a single crane, placing the cables can be accomplished in a variety of ways. All lifting cables can come to a single point, or a carrying beam can be used, which opens up a whole new array of options. The number of lifting points used is a major factor in determining cable placement. Other factors influencing cable placement include the size of the slab, the prestressing, and the strength of the slab, etc. These variables are outlined in Table 4: Lifting Variables.

Table 4: Lifting Variables

Lifting Variables		
Variable	Range	
Length of Slab	34 ft	78 ft
Prestressing Load	20 kips	31 kips
Concrete Strength	4 ksi	8 ksi
Lifting Points	4	22
Carrying Beam	None	Two

SAP2000 Lifting Analysis for Model

A SAP2000 analysis was completed on the Gilford Bridge model to determine what kind of stresses would be seen in the slab when lifted from all 14 leveling devices using a carrying beam. The maximum tensile stress was found to be 143psi. A subsequent SAP2000 analysis was then performed lifting from only 4 locations within PCI equation tolerance. This analysis produced a maximum tensile stress of 531psi.

All varied length models were analyzed under a series of different lifting locations and number of lifting points. The maximum tensile stress was taken from the SAP2000 analysis and combined with the calculated prestress compression, taking into consideration losses at the cross section where blockouts exist.

Table 5: Varied Length Models Output summarizes the final stresses from a number of different analyses.

When it comes to lifting the slabs, the most conservative method is the way to prevent cracking. Lifting from all leveling devices available is the concluded recommended way, but ultimately it will come down to the time of lifting in the curing period of the concrete, the length of the slab, the fabricator, and the design engineer. If the engineer feels confident that the slab can be lifted from PCI's eight point lift configuration, additional lifting devices must be cast into place. Otherwise, a lifting configuration must be achieved that utilizes the existing leveling devices.

Table 5: Varied Length Models Output

Stresses from Dead Load Lifting						
Slab Length (ft)	# Lifting Points	Carrying Beam?	Max Tensile Stress (psi)	Prestressing Stress (psi)	Final Stress (psi)	
34	8	no	294	397	(103)	*Compression*
34	10	yes	50	397	(347)	*Compression*
48	8	no	500	397	103	*Tension*
48	12	no	315	397	(82)	*Compression*
48	12	yes	314	397	(83)	*Compression*
48	14	yes	143	397	(254)	*Compression*
64	8	no	594	397	197	*Tension*
64	16	no	409	397	12	*Tension*
64	16	yes	305	397	(92)	*Compression*
78	8	no	738	397	341	*Tension*
78	12	no	970	397	573	*Tension*
78	16	no	343	397	(54)	*Compression*
78	20	yes	306	397	(91)	*Compression*

For Gilford, it just so happens to work out that the leveling devices are spaced that they come very close to what PCI defines for an eight point lift. With full strength concrete and the prestressing compression developed within the slab, an eight point lift should be an acceptable method if minimal cracking is acceptable. At stripping, lifting from the full 14 leveling device locations is still recommended.

Transportation Options

The final major hurdle to overcome when using long, minimal depth, crowned slabs is transportation. A few different methods were analyzed in attempt to overcome this challenge.

To analyze and test one method, the slab model was placed on a trucking frame connected to an extendable bed that a conventional truck could tow. The extendable bed trucking option allows the slab to be placed on a rigid platform. The slab can be placed on planking so as not to induce a load different from the design installation loading. This kind of truck extension can be seen in Figure 20: Extendable Truck Bed

Figure 20: Extendable Truck Bed

Other types of trucking methods that were considered were a tractor and dolly system as well as a conventional flatbed truck. The truck and dolly option allows for longer members, yet bending, tension, and torsion loads during trucking must be considered. This transportation option secures a member on the dolly, and at the truck the support is equivalent to a ball joint. This requires the sufficient torsional rigidity in the member to resist torsion along its length when the dolly and truck are not on a horizontal surface but at different slopes. However, the tractor and dolly system isn’t the most financially feasible option and a standard flat bed is too short for the Gilford Bridge slabs to fit on.

The recommended trucking method is the extendable bed with a trucking frame that the slab will rest on. The frame will be connected to the bed with a ball joint connection over the location where the bed attaches to the truck, and a pin type connection at the rear of the bed. This will allow the slab to move somewhat independently from the bed for situations such as turning tight corners or encountering super elevated curves.

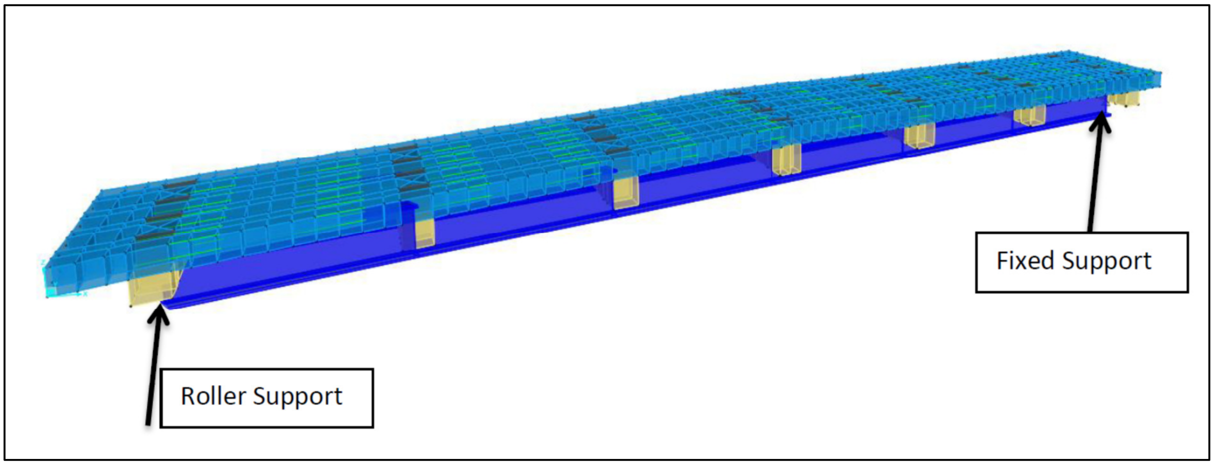

Figure 21: Trucking Frame Supports

Simulate Bridge Girders

The trucking frame was designed to simulate the supporting bridge girders that the slab was designed to be supported on (see Figure 21). This was also found to be the best way to provide stability during transportation and decrease opportunities for stress cracking. The trucking frame is outfitted with wood shim supports, spaced at the same distance as the bridge girders. Through analysis with the varied length SAP2000 models it was determined that this would only be necessary for slabs that exceed 48 ft in length, from tip to tip of slab. For slabs less than this, which would mean an unconventional bridge width, supporting at fifth points or standard PCI lifting point locations would serve as viable options on a standard flatbed truck. All slabs were analyzed with a 1.3 impact factor for transportation. The dead load was increased by 30% to mimic the forces of going over bumps and cracks in the roadway. This is slightly less than the current LRFD 3.6.2.1 of 33%.

The steel supports extend the full transverse width of the slab, wood shims will be placed on top of these supports; the steel supports will rest on two steel girders connected and spanning the full length of the extendable bed truck. Holes drilled in the wood shims bearing on the steel supports allow space for the leveling devices to allow the devices to be used as tie down mechanisms.

Shimmed Support to Account for Crown and Deflection

Since the slabs will be cast with a crown, the shimmed supports cannot all be the same height. Therefore, the shimmed supports will have to be appropriately sized to accommodate the dead load deflection and the crown of the slab. The wood shims will have a base height of 12 inches for a slab that is 48 feet long. The shim located in the center of the slab will have the greatest depth of all the supports to accommodate the crown and deflection (see Figure 22).

The shimmed supports are made out of wood for a number of reasons. It is much more economical to use wood rather than steel W shapes to support the slab in the lateral dimension. Secondly, wood can be cut, shaped, drilled, etc., much more easily than steel or concrete. In addition, these wood shims can also be reused and modified to the specific project.

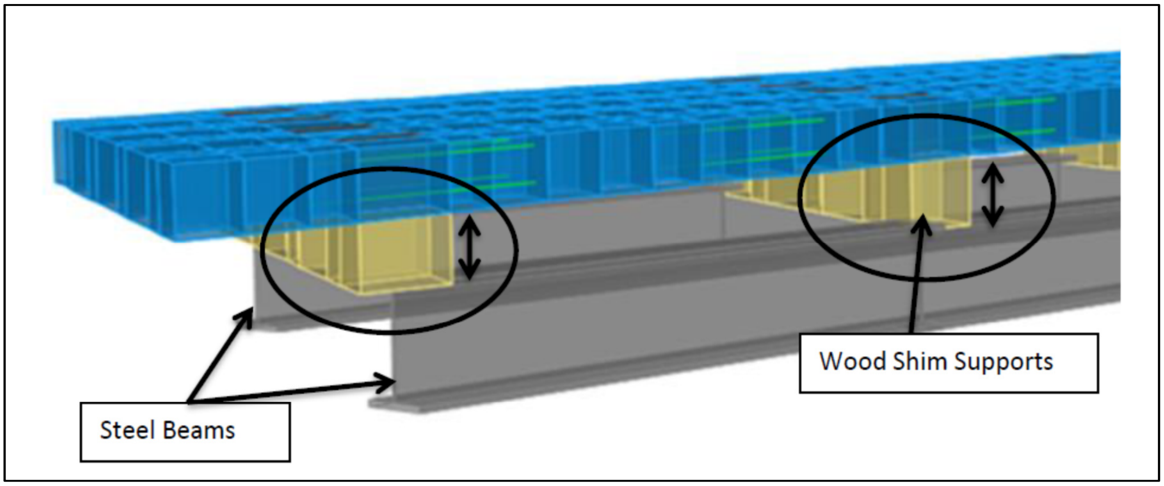

Figure 22: Wood Shims

The deflection of the girder under its self-weight can be calculated using a simple uniform load equation. The deflection caused by the dead load of the slab can then be superimposed on the deflection under the self-weight. It is assumed the weight of the wood shimmed supports is negligible. The weight of the concrete slab is condensed into point loads to calculate deflection. The number of point loads will depend on the actual number of bridge girders. The point load will be calculated by using the typical weight of concrete,150 pcf, and multiplying it by the slab volume.

To find the individual loads for each support, the tributary width must be calculated for each individual support. For the basis of this research, a girder spacing of 7'- 4" was used. A sample of the calculated shim heights for the 1/8 scale laboratory model can be seen in Table 6.

This method for trucking can be used for any length of slab that an extendable bed can accommodate. Beyond that, the frame can be modified slightly to accommodate a tractor and dolly system setup to transport the slabs. The same design and analysis methodology will still work for any length a steel W section can carry.

Table 6: Shim Height Calculation for 1/8 Scale Model

	Girder Deflections				
Slab Information					
length =	81	in			
width =	12.5	in			
depth =	1.25	in			
Volume =	0.732	ft^3			
Beam Information					
weight =	109.86	lbs			
w =	0.000646	k/in		1" square tubing 16 gauge	
E =	29000	ksi		1	0.083
I =	0.03559	in^4		0.87	0.048
L =	85	in		I =	0.036
Leveling Screw 1				Slope Heights	**Shim Heights (in)**
x =	10.625	in			
Δ @ x =	**0.165**	in	+	0.78125	**0.95**
Leveling Screw 2					
x =	21.25	in			
Δ @ x =	**0.303**	in	+	1.09	**1.40**
Leveling Screw 3					
x =	31.875	in			
Δ @ x =	**0.394**	in	+	1.4375	**1.83**
Leveling Screw Middle					
x =	42.5	in			
Δ @ x =	**0.426**	in	+	1.75	**2.18**
max deflection =	*0.4255907*	*Check*			

Lab Model of Trucking Structure

To test the methodology by which the deflections of the steel support members are calculated, a lab model of the trucking frame was built (see Figure 23). The W sections were modeled with 1” square steel tubes, 48 ksi steel, welded together in a parallelogram at the 23° skew. A thin walled section was specifically chosen to test the deflection calculations. It was intended to deflect a significant amount under the dead load of the lab model of the Gilford slab. Wood shim supports heights were then calculated, cut to size, and attached to the frame. A layer of foam was applied to the top side of the wood shims to soften the resting surface for the concrete slab.

The experiment proved to be successful. The slab was lifted, using a 14-point lift with a carrying beam in the lab, and slowly set on the trucking structure model. The wood shims supported the slab evenly and the frame deflected within 10% of calculated amount. This test then concluded this stage of research for the fabrication, handling, and transporting of the crowned bridge deck slabs.

Figure 23: Lab Slab Trucking Frame

Conclusion

This research illustrates that the fabrication, handling, and transportation of full width crowned bridge deck slabs is a feasible option for replacement. With using the methods discussed the slabs can be cast by conventional prestressing fabricators with minor adjustments made to the form beds. The slabs can then be handled on site safely using the lifting methods described and then transported on a reusable frame that can easily be attached to an extendable truck bed, pulled by a standard truck tractor.

Replacing just the bridge deck with this method of ABC will save money and considerable bridge down time. The entire process can be completed in a matter of days rather than months. This saves cost for detours, managing traffic, labor time, and significantly minimizes the time of intrusion on neighbors. This new method of rapid bridge deck replacement takes advantage of all that pre-cast concrete has to offer with a few minor accommodations to make it safe, feasible, and cost efficient.